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DEWATERING OF BENTHIC DEPOSITS ON SOILS

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DEWATERING OF BENTHIC DEPOSITS ON SOILS

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ABSTRACT

Benthic deposits in our nation's rivers, lakes, reservoirs, and estuaries are producing very difficult problems of pollution including eutrophication. The only solution for the majority of these problems is the removal of these deposits. However, this does not represent a final solution. Disposal of the voluminous dredgings may result in surface and ground water pollution at the disposal site. Such pollution can be controlled by dewatering the deposits prior to final disposal. Gravity dewatering of these highly organic, compressible materials on prepared soils appears practicable. However, optimum design and operation of the dewatering sites have been hampered by the paucity of loading rate criteria, i. e., weight of solids dewatered per unit time and area.

The authors have made a theoretical analysis of gravity dewatering of dredging slurries and dilute organic suspensions. A three-parameter equation was developed relating dewatering rate to the solids content, specific resistance, and compressibility coefficient of the suspensions. All three parameters are readily measured in the laboratory. The theory was validated by extensive bench scale tests, and it was determined that an accurate, dependable analytical method is now available to determine dewatering rates of sludges on soils. It is felt that a rational basis for design and operation of dredging dewatering sites now exists.

I. INTRODUCTION

Background of Problem

Benthic deposits are defined as the deposits on the bottom of rivers, lakes, estuaries, and impoundments. Benthic deposits are usually differentiated from deposits such as sandbars in that they are normally biologically and chemically active. As such, they may exert a significant influence on the presence or absence of dissolved constituents in the overlying water. The biological and chemical activity of benthic deposits derives from their origin; they may be derived from dead algae which have settled from above, leaves which have been washed or blown into the water; and solids originating from wastewaters, either treated or untreated. In rivers sludge banks, or benthic deposits, from discharges of untreated sewage may attain a thickness of several feet prior to scouring and redeposition by floods. In impoundments and lakes volume reduction of the deposits is achieved through compaction, while biological decomposition to stable end products is underway.

If sufficient oxygen is present in the overlying waters, surface portions of benthic deposits may be aerobic. However, interior portions may consume oxygen at a more rapid rate than it diffuses into the mass, so that anaerobic conditions extend from near the surface to the interior of the deposits. Laboratory analyses at the University of Massachusetts have shown that an oxidized microzone extends less than an inch into the sludge when the overlying water was aerobic[1]. Oxidation-reduction reactions are active near this zone. Hutchinson[2] has pointed out that the presence or absence of an oxidized microzone plays a significant role on the chemical exchange rate between benthic deposits and the overlying waters.

Chemical exchange is enhanced or impeded by the oxygen content of the overlying waters. In lakes and impoundments this is subject to diannual cycles coinciding with the overturning of the lake and impoundment contents. Two constituents, nitrogen and phosphorous, are especially important because of their role as the limiting nutrients which retard algal blooms when they are unavailable in sufficient concentrations[3]. Benthic deposits appear to be a reservoir for both nitrogen and phosphorous once these elements have been added by surface runoff or wastewaters. This reservoir may release these elements to the overlying waters under reducing conditions. If released, algal blooms may be promoted, resulting in rapid eutrophication with accelerated aging of the lake.

Removal of the benthic deposits is the only practical long term solution which will reverse the aging process. If removed, disposal of the benthic deposits is a problem. Land disposal permits use of the dredged material for fill, although some highly organic deposits may be useful as a soil conditioner. However, prior to such usage volume reduction is necessary, for if removed on a large scale by hydraulic dredging, the resulting slurry will be high in water and low in solids content. This suggests a dewatering, or solids-liquid separation, process will be necessary to concentrate the solids.

Dewatering organic sludges has been practical for many years as a wastewater treatment operation. Studies by the City of Chicago show costs of dewatering, drying and disposal varies from $12/ton to $60/ton on a dry solids basis with the higher cost for mechanical dewatering methods and the lower cost for dewatering on sand beds with subsequent land disposal of the solids[4]. However, design data for sand dewatering beds is limited and empirical formulations or rules of thumb for design are frequently used. A rational design method for dewatering slurries on sand beds is formulated

in subsequent sections of this paper.

Classification of Sludges

Sludges are characterized by a high water content and a low solids content prior to concentration. For example, a wet sludge weighing 1000 pounds when removed may weigh only 30 pounds after it has been dewatered and air dried. This emphasizes the immense importance of the dewatering step in reducing the volume and weight of a sludge to manageable proportions.

Dewatering on beds of granular media is influenced by properties of the sludge and properties of the media. Coarse media may promote rapid dewatering for a short time, but if solids are carried into the bed, clogging may result in attenuated bed life. Also, characteristics of the sludge may limit the dewatering rate if a tight cake is formed on the surface of the dewatering bed.

From the previous discussion it is seen that filter characteristics such as its grain size and permeability are important, as well as sludge chacteristics such as its permeability, solids content, and penetrating ability. As benthic deposits may be high in organics and flocculant solids, their sludges are found to be compressible. Compression during dewatering alters the permeability. This further complicates development of dewatering theory. However, as shown subsequently, use of the concept of specific resistance and coefficient of compressibility of a sludge cake permits formulation of a dewatering model applicable to compressible materials, such as benthic deposits.

II. THEORY OF DEWATERING OF COMPRESSIBLE MATERIALS

The elements of filtration theory derive from the concepts of fluid flow through closed conduits. If a Newtonian fluid in laminar flow through a circular pipe is assumed subject to pressure, gravity, and viscous forces; the equation of Hagen-Poiseuille holds. It is generally in the form

$$\Delta p = \frac{32 \mu \bar{v} L}{D^2} \qquad 2\text{-}1$$

with Δp = pressure drop along the length of the pipe

μ = dynamic viscosity of the fluid $[F][T][L^{-2}]$

$\bar{v}$ = average velocity of the fluid $[L][T^{-1}]$

L = length [1]

D = diameter of pipe $[F][L^{-2}]$

It is here more convenient to express pressure drop in terms of headloss. A dimensionally correct conversion is

$$\Delta p = \rho g h_f$$

with ρ = density of the fluid $[M][L^{-3}]$

g = acceleration of gravity $[L][T^{-2}]$

h_f = equivalent height of fluid [L]

Thus

$$\rho g h_f = \frac{32 \mu \bar{v} L}{D^2}$$

or

$$h_f = \frac{32 \mu \bar{v} L}{\rho g D^2} \qquad 2\text{-}1a$$

Equation 2-4a may be modified for non-circular cross-sections by use of the concept of hydraulic radius (R_h). The hydraulic radius is defined as the ratio of cross-sectional area A to the wetted perimeter of the conduit P. For circular cross-sections this is

$$R_h = \frac{A}{P} = \frac{\pi D^2/4}{\pi D} = \frac{D}{4}$$

R_h has the unit of length. It allows expression of the Hagen-Poiseuille equation as

$$h_f = \frac{2\mu \bar{V} L}{\rho g R_h^2} \qquad 2\text{-}1b$$

For the more general case of non-circular cross-sections, Equation 2-1b should be modified to

$$h_f = \frac{K\mu \bar{V} L}{\rho g R_h^2} \qquad 2\text{-}1c$$

with K a dimensionless variable, dependent on the shape of the cross-section.

FLOW THROUGH POROUS MEDIA

Flow through porous media is generally represented as laminar flow through a series of interconnected pores of varying cross-section. An initial assumption of a media composed of identical particles is also made.

If N represents the number of particles in a sample of L length and cross-sectional area A', then the wetted conduit surface P over unit length may be written as

$$P = \frac{N S_p}{L} \qquad 2\text{-}2$$

with S_p = surface area of a single particle [L^2]

The cross-sectional flow area A is some fraction of the total cross-sectional area A'. This fraction is the porosity ϵ :

$$\epsilon = \frac{V_v}{V_{TOT}} \qquad 2\text{-}3$$

with V_v = volume of voids in the sample [L^3]

V_{TOT} = total volume of sample, or the volume of voids and solids [L^3]

If the particles are each of volume V_p, then $N \cdot V_p$ represents the volume of solids, allowing one to write

$$V_{TOT} = V_v + N V_p$$

Thus

$$\epsilon = \frac{V_v}{V_{TOT}} = \frac{V_v}{V_v + N V_p}$$

or rearranging,

$$V_v = \frac{\epsilon N V_p}{1 - \epsilon} \qquad 2\text{-}4$$

Substitution of equation 2-2 into

$$R_h = \frac{A}{P}$$

yields

$$R_h = \frac{AL}{N S_p}$$

Noting that the product A·L equals the volume of voids in the sample (V_V), combination of Equations 2-4 and 2-5 yields

$$R_h = \frac{AL}{N S_p} = \frac{V_v}{N S_p} = \frac{\epsilon N V_p}{1 - \epsilon} \cdot \frac{1}{N S_p}$$

or

$$R_h = \frac{\epsilon}{1-\epsilon} \cdot \frac{V_p}{S_p}$$
$$= \frac{\epsilon}{1-\epsilon} \cdot \frac{1}{S_o} \qquad 2\text{-}6$$

with S_o = specific surface or ratio of area of particle to volume $[L^{-1}]$.

The velocity $\bar{v}$ through the pores of the sample is generally not readily determinable. Instead of $\bar{v}$, a velocity called the superficial velocity, $\bar{v}_s$, is used. It may be thought of as the velocity of flow if the particles are considered non-existent. The equation of continuity defines $\bar{v}_s$:

$$\bar{V}_s A' = \bar{V} A$$

or

$$\bar{V} = \bar{V}_s \frac{A'}{A}$$

Multiplication of the right hand side of this equation by L/L = 1 yields

$$\bar{V} = \bar{V}_s \frac{A'L}{AL}$$

The product A'L is again the volume of voids in the sample, whereas A''L is the total volume; thus

$$\bar{V} = \bar{V}_s \frac{V_{TOT}}{V_v}$$

or

$$\bar{V} = \frac{\bar{V}_s}{\epsilon} \qquad 2\text{-}7$$

as V_V/V_{TOT} is the porosity .

Substitution of Equations 2-6 and 2-7 into Equation of 2-1c follows:

$$h_f = \frac{K \mu \bar{V} L}{\rho g R_h^2}$$
$$= \frac{K \mu L}{\rho g} \cdot \frac{\bar{V}_s}{\epsilon} \Big/ \left[\left(\frac{\epsilon}{1-\epsilon}\right)\frac{1}{S_o}\right]^2$$

which simplifies to

$$h_f = \frac{\mu L \bar{v}_s K}{\rho g \epsilon} \left[\frac{1-\epsilon}{\epsilon S_o}\right]^2 \qquad 2\text{-}8$$

This equation is frequently written as

$$\bar{v}_s = \frac{\rho g h_f \epsilon}{L K} \left[\frac{\epsilon}{(1-\epsilon) S_o}\right]^2$$

The end terms

$$\frac{\epsilon}{K}\left[\frac{\epsilon}{(1-\epsilon)S_o}\right]^2$$

are recognized as intrinsic properties of the porous medium, and are thus given the substitutive symbol k, the intrinsic permeability. The reciprocal of k is R', the filter resistance. It has units of $[L^{-2}]$. Equation 2-8 may therefore be written as

$$h_f = \frac{\mu L \bar{v}_s R'}{\rho g} \qquad 2\text{-}9$$

with

$$R' = \frac{K}{\epsilon}\left[\frac{(1-\epsilon)S_o}{\epsilon}\right]^2 \qquad 2\text{-}10$$

CARMEN-KOZENY RELATIONSHIP

The variables involved in Equation 2-10

$$R' = \frac{K}{\epsilon}\left[\frac{(1-\epsilon)S_o}{\epsilon}\right]^2$$

bear closer scrutiny. It is clear that if the sample consists of identical geometrically describable particles, the value of S_o may be directly calculable. The values of K and ϵ, however, depend on the packing or placement of the particles. Empirical corrections are applied, resulting in

$$R' = \frac{K'}{\epsilon}\left[\frac{(1-\epsilon)S'}{\epsilon}\right]^2 \qquad 2\text{-}10a$$

with S' defined as a shape factor. Theoretically K' is a function of particle shape and orientation (i.e., the shape of cross-section available for flow). Grace[2] reports that for particles of diameters $< 5\mu$, k' is a variable, due to compression of the small particles. S', the shape factor, is of course, extremely variable, particularly for flocculent particles.

Rewriting of Equation 2-9 using Equation 2-10a

$$h_f = \frac{\mu L \bar{v}_s R'}{\rho g}$$

leads to

$$h_f = \frac{\mu L \bar{v}_s K'}{\rho g \epsilon} \left[\frac{(1-\epsilon) S'}{\epsilon} \right]^2 \qquad 2\text{-}10b$$

which is the Kozeny-Carmen relationship[3]. It may be modified for nonuniform materials by reference to a particle size analysis, allowing average values for particle area and volume to be used.

FILTRATION OF COMPRESSIBLE MEDIA

It has been repeatedly proven that Equation 2-10b does not apply to the filtration of very fine suspensions such as organic sludges[4]. This is true for two reasons:

1. The suspended materials are of a gelatinous nature and may flocculate when brought in close proximity to each other.
2. The extremely fine size creates a large headloss through a short distance. The loss is in the form of viscous drag over the particle surfaces resulting in particle deformations and reduction of void sizes.

Hence it is seen that the term R' in Equation 2-9 reflecting the influence of shape, size, and orientation of particle, and the void ratio, is a variable for filtration of fine suspensions. Further analysis requires the assumption

of a gradual increase in filter resistance with duration of filtration. A later assumption will be the dependence of filter resistance on the pressure applied across the sample.

Assume a filter material of known and constant resistance which will serve as a supporting media to the filter cake formed on its surface during filtration. The resistance of this cake may be called $R'_f[L^{-2}]$ and the depth, L_f. It follows that Equation 2-9

$$h_f = \frac{\mu L \bar{v}_s R'}{\rho g} \qquad 2\text{-}9$$

may include the effects of the supporting filter by writing

$$h_f = \frac{\mu}{\rho g}\left(\bar{v}_s L R' + \bar{v}_s L_f R'_f \right) \qquad 2\text{-}11$$

with $\bar{v}_s$ = superficial velocity in supporting filter.

The superficial velocities are best rewritten in terms of the filtrate volume flow rate dV/dt $[L^3][T^{-1}]$. Thus:

$$\bar{v}_s = \frac{1}{A}\frac{dV}{dt}$$

and

$$\bar{v}_s' = \frac{1}{A}\frac{dV}{dt}$$

Hence Equation 2-11 reads

$$h_f = \frac{\mu}{\rho g A}\frac{dV}{dt}\left(L R' + L_f R_f' \right) \qquad 2\text{-}11a$$

which reorganizes to

$$\frac{dV}{dt} = \frac{\rho g A h_f}{\mu (L R' + L_f R_f')} \qquad 2\text{-}11b$$

Carmen[3] introduced the concept that the length L of the compressible

filter cake was proportional to the volume of filtrate:

$$L = v_c \, V/A$$

with v_c = volume of cake deposited per unit volume of filtrate (dimensionless)

This leads to

$$\frac{dV}{dt} = \frac{\rho g A h_f}{\mu\left(\frac{v_c V R'}{A} + L_f R'_f\right)} \qquad 2\text{-}11c$$

However, the nature of compressible cakes makes it illogical to measure volume. More likely is the measurement of weight. The term c, or weight of dry cake solids per unit volume of filtrate is thus introduced. Dimensions of c are $[F][L^{-3}]$. This necessitates a new definition of filter resistance, R, by setting

$$v_c R' = c R$$

R, entitled the specific resistance, is in units of $[L][F^{-1}]=[T^2][M^{-1}]$

Equation 2-11c can be reformulated then to

$$\frac{dV}{dt} = \frac{\rho g A h_f}{\mu\left(\frac{cRV}{A} + L_f R'_f\right)}$$

$$= \frac{\rho g A^2 h_f}{\mu(cRV + AL_f R'_f)} \qquad 2\text{-}11d$$

This equation is the basis for work done by Coackley and Jones[5], utilizing the concept of specific resistance to calculate vacuum filter performance in dewatering wastewater sludges.

SPECIFIC RESISTANCE

The specific resistance of a material varies with depth, being a maximum at the supporting filter interface. For shallow depths, however, R varies

slightly, and thus an average may be used. The value of R is determined in the laboratory by constant pressure filtration: a constant pressure or headloss, h_f, allows integration of Equation 2-11d.

$$\frac{dV}{dt} = \frac{\rho g A h_f}{\mu\left(\frac{cRV}{A} + L_f R'_f\right)} \qquad 2\text{-}11d$$

$$\int_0^t \rho g A h_f \, dt = \int_0^V \frac{\mu c R V \, dV}{A} + \int_0^V \mu L_f R'_f \, dV$$

$$t = \frac{\mu c R V^2}{2 \rho g A^2 h_f} + \frac{\mu L_f R'_f V}{\rho g A h_f} \qquad 2\text{-}12$$

COEFFICIENT OF COMPRESSIBILITY

It must be emphasized that the value of R is dependent not only upon the sludge characteristics but on the pressure at which the test is run. It is known from experimental analysis that a relation between R and Δp exists[2]. It is in the form

$$R = a_o + a' (\Delta p)^{\sigma} \qquad 2\text{-}13$$

or using headloss

$$R = a_o + a' (\rho g h_f)^{\sigma} \qquad 2\text{-}13a$$

with a_0, and a' and σ constants for a given sample, and independent of pressure. σ is commonly known as the cake compressibility factor, and a' the cake constant. Several determinations of R for various pressures may be plotted on bi-logarithmic paper, the slope of the line of best fit revealing the cake compressibility factor. Generally a_0 is neglected,

simplifying Equation 2-13a to

$$R = a'(\rho g h_f)^{\sigma} \qquad 2\text{-}14$$

Equation 2-14 proves dimensionally incorrect. It may be modified for dimensional correctness by writing

$$R = a' \left(\frac{\rho g h_f}{\Delta P a = 1} \right)^{\sigma} \qquad 2\text{-}15$$

which therefore describes a' as the specific resistance $[T^2][M^{-1}]$ at a pressure of unity $[F][L^{-2}]$. Equation 2-15 does not, however, differ numerically from Equation 2-14.

GRAVITY DEWATERING

Application of the theory and concept of specific resistance to gravity drainage of organic sludges on porous media is most promising. Assume a sludge is applied on a dewatering bed of sand or other porous material. Referring back to Equation 2-11d

$$\frac{dV}{dt} = \frac{\rho g A^2 h_f}{\mu (c R V + A L_f R'_f)} \qquad 2\text{-}11d$$

the term dV/dt may be rewritten in terms of the head as the rate of drop of the sludge surface by drainage

$$\frac{dV}{dt} = -A \frac{dH}{dt}$$

with H = the head [L]

The terms referring to the supporting media may be ignored, as sludge is vastly higher in resistance than permeable soils[6].

$$-A \frac{dH}{dt} = \frac{\rho g A^2 h_f}{\mu c R V}$$

But h_f is none other than H, assuming of course, that resistance in the filter is minimal. Hence, with cancellation of A

$$\frac{dH}{dt} = \frac{-\rho g A H}{\mu c R V} \qquad 2\text{-}16$$

This equation may be written for a cumulative drop from height H_0 at t=0 upon substitution of $A(H_0-H)$ for V:

$$\frac{dH}{dt} = \frac{-\rho g A H}{\mu c A (H_0-H) R} \qquad 2\text{-}17$$

Noting that

$$R = a'(\rho g h_f)^{\sigma} \qquad 2\text{-}14$$

or here

$$R = a'(\rho g H)^{\sigma}$$

Equation 2-17 can be rewritten as

$$\frac{dH}{dt} = \frac{-\rho g H}{\mu c (H_0-H) a' (\rho g H)^{\sigma}}$$

reorganized to

$$dt = -a' \mu c (H_0-H)(\rho g H)^{\sigma-1} dH \qquad 2\text{-}18$$

This equation may be integrated from $H = H_0$ at $t = 0$:

$$\int_0^t dt = -\int_{H_0}^{H} a' \mu c (H_0-H)(\rho g H)^{\sigma-1} dH$$

$$= -a' \mu c (\rho g)^{\sigma-1} \left(\int_{H_0}^{H} H_0 H^{\sigma-1} dH - \int_{H_0}^{H} H^{\sigma} dH \right)$$

which yields

$$t = -a' \mu c (\rho g)^{\sigma-1} \left[\frac{H_o H^{\sigma}}{\sigma} - \frac{H^{\sigma+1}}{\sigma+1} \right]_{H_o}^{H}$$

$$= -a' \mu c (\rho g)^{\sigma-1} \left(\frac{H_o H^{\sigma} - H_o^{\sigma+1}}{\sigma} + \frac{H_o^{\sigma+1} - H^{\sigma+1}}{\sigma+1} \right) \qquad 2\text{-}19$$

Equation 2-19 allows determination of the drainage rate of a sludge on porous material, provided the initial sludge solids content is evenly distributed. A further limitation is the assumption of filtration in the absence of consolidation, whereby compaction creates a pressure gradient near the top in the upward direction. Shallow layers, also dilute suspensions of sludges do not present this complication.

Benthic sludges are known frequently to settle out rapidly when allowed to dewater. A modification of Equation 2-31a may be developed for the case of a supernatant draining on a settled sludge layer. Another analysis of great potential interest is the drainage of a sludge on a previously applied sludge.

III. EXPERIMENTAL PROCEDURE AND VERIFICATION

Sampling

Benthic deposits were removed by an Ekman clam-shell bottom dredge. Samples were all taken in the vicinity of Amherst from two small ponds and a water supply reservoir. These were in turn characterized as to depth of water and depth of deposit as follows:

Source	Depth of Water Feet	Depth of Deposit Feet	Drainage Area	Water Surface Area
Atkins Reservoir	8	0.1	800 acres	40 acres
Campus Pond	5	3	130 acres	12 acres
Spring Pond	6	3	60 acres	1 acre

Campus Pond was unique in that it is in the midst of a developed area and had been dredged in the past decade. Spring Pond has no surface influent or effluent. Both Spring Pond and Atkins Reservoir lie in protected forests. All samples appeared relatively homogenous, made up of dense slurries with a few clumps of vegetation which were readily disposed of. All samples were refrigerated previous to testing. A shallow zone of supernatant developed after two or three days.

The samples were classified according to standard methods of determination of water content on a wet-weight basis[1]. Also weight loss at T = 600°C was determined, and expressed as the residue remaining at that temperature in per cent of the residue remaining at 103°C. Results are found in Table 3-1.

Table 3-1: Classification of Deposits

Source	~~Water~~ Solids Content (% wet-basis)	Residue @ 600°C (% of residue @ 103°C)	Specific Resistance (sec^2/gm)	Coefficient of Compressibility
Atkins Reservoir	5.19	79.8	7.7×10^8	0.82
Campus Pond	42.75	91.1	1.26×10^8	0.62
Spring Pond	6.88	62.7	11.0×10^8	0.84

Specific Resistance

The method of determination of specific resistance has to date not been standardized. It may be expected that the method chosen will influence the results, however, such influence is thought minor.

Basically, the specific resistance is determined by applying a constant vacuum across a sludge cake, and measuring filtrate volume and time at approximately 30 second intervals for a total duration of about seven minutes [2]. Figure 3-1 portrays the necessary equipment.

A 100 ml sample of sludge is poured onto a wetted Whatman #5 filter paper. At time zero the vacuum is applied and readings are taken as previously described. At the end of the test, the cake sample is weighed and dried, and a wet-weight water content is determined.

An easy method for data reduction has been demonstrated by Coackley and Jones[3]. Referring back to Equation 2-12, one notes that it may be reformulated in terms of t/V versus V:

$$\frac{t}{V} = \frac{\mu c R}{2 \rho g A^2 h_f} V + \frac{\mu L_f R'_f}{\rho g A h_f} \qquad 3\text{-}1$$

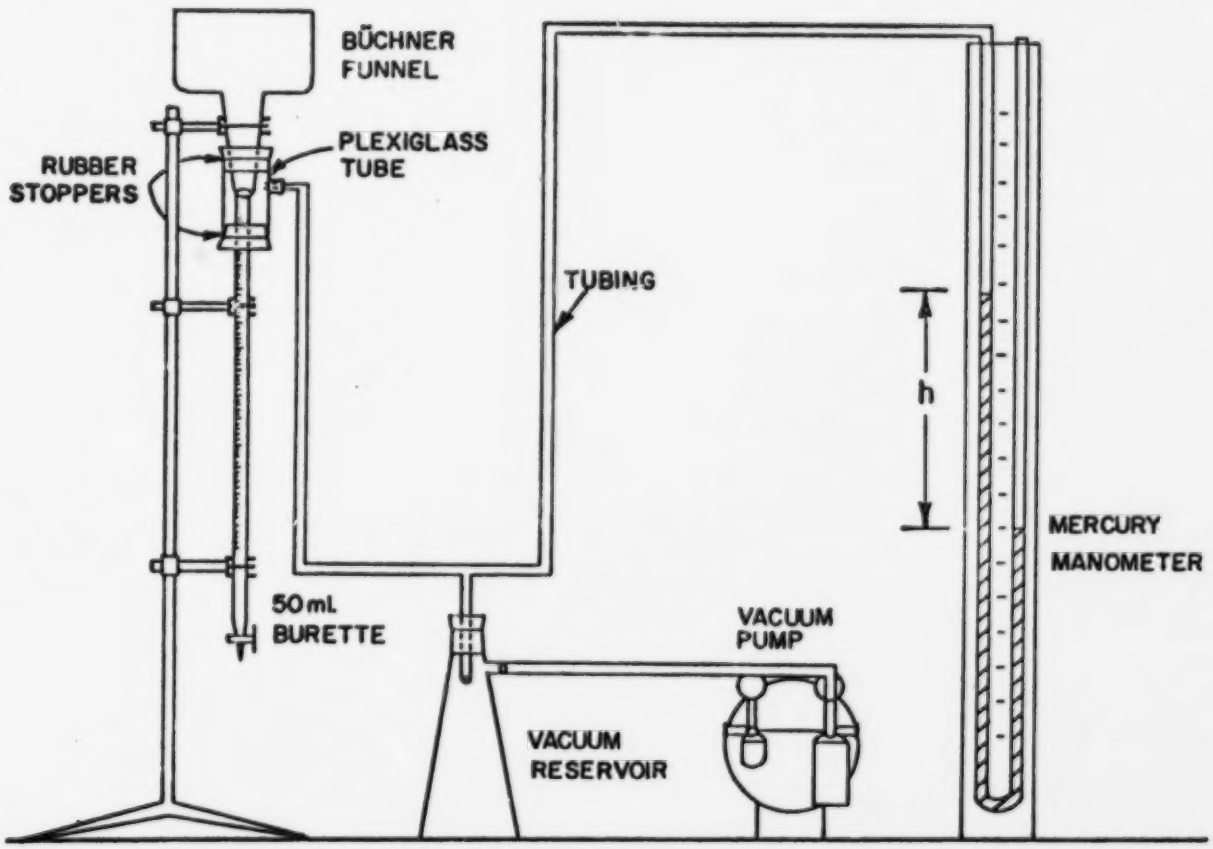

Figure 3-1: Apparatus to determine the specific resistance of compressible sludges.

A plot of t/V versus V (Figure 3-2) will yield a straight line, the slope of which is seen to equal the fractional coefficient in Equation 3-1 (see Figure 3-2). The specific resistance will then be

$$R = \frac{2\rho g A^2 h_f}{\mu c} (slope) \qquad 3\text{-}2$$

The area A of the filter area and the density ρ , and the dynamic viscosity μ of the fluid are readily determinable. The vacuum, h_T, was set and known. There remains the determination of c, the dry weight of cake solids per unit volume of filtrate.

The volume of filtrate that occurs during testing, assuming no solids loss, is

$$V = (W_{TOT_o} - W_{TOT_f})/\rho g \qquad 3\text{-}3$$

with V = volume of filtrate $[L^3]$

W_{TOT_o} = weight of unfiltered sludge [F]

W_{TOT_f} = weight of filtered cake [F]

ρg = weight-volume conversion factor $[F][L^{-3}]$

(ρ refers to filtrate)

The weight of dry cake solids per unit volume of filtrate is $c = W_{TS}/V$

which may be written

$$c = \frac{W_{TS}\,\rho g}{W_{TOT_o} - W_{TOT_f}}$$

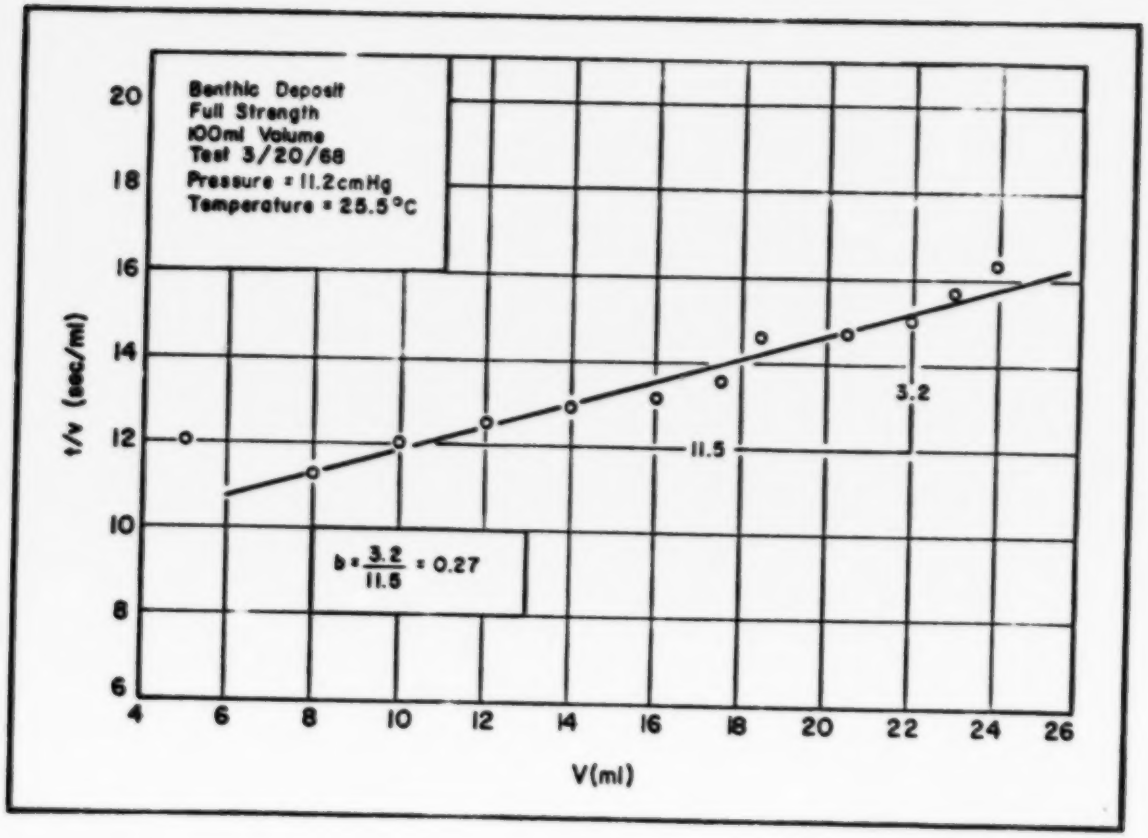

Figure 3-2: Sample plot to determine specific resistance for deposits from Campus Pond.

However, W_{TS} can be expressed in terms of the initial wet-basis water content

$$W_{TS} = W_{W_o}\left(\frac{100}{\theta_o} - 1\right)$$

with W_{W_o} = initial water content of sludge [F]

θ_o = initial wet-basis water content [%]

Hence

$$c = \frac{W_{W_o}\, \rho g \left(\frac{100}{\theta_o} - 1\right)}{W_{TOT_o} - W_{TOT_f}} \qquad 3\text{-}3a$$

Equation 3-3a was used to calculate the value of c, which in turn was placed in Equation 3-2 for eventual calculation of R. Results are shown in Table 3-1. As emphasized in Section II, the value of R is dependent on pressure. The literature generally refers to a pressure of 15 in. Hg (= 38.1 cm). This convention is followed here.

Coefficient of Compressibility

The dependence of R on pressure as expressed by Equation 2-14 can be experimentally determined by a series of different R determinations or a number of different constant pressures. As R varies with h_f to the σ power, σ being the coefficient of compressibility, it is evident that a bilogarithmic plot of R versus h_f will yield a slope of value σ. This procedure was used, an example is given in Figure 3-3, and the coefficient for the sludges is to be found in Table 3-1.

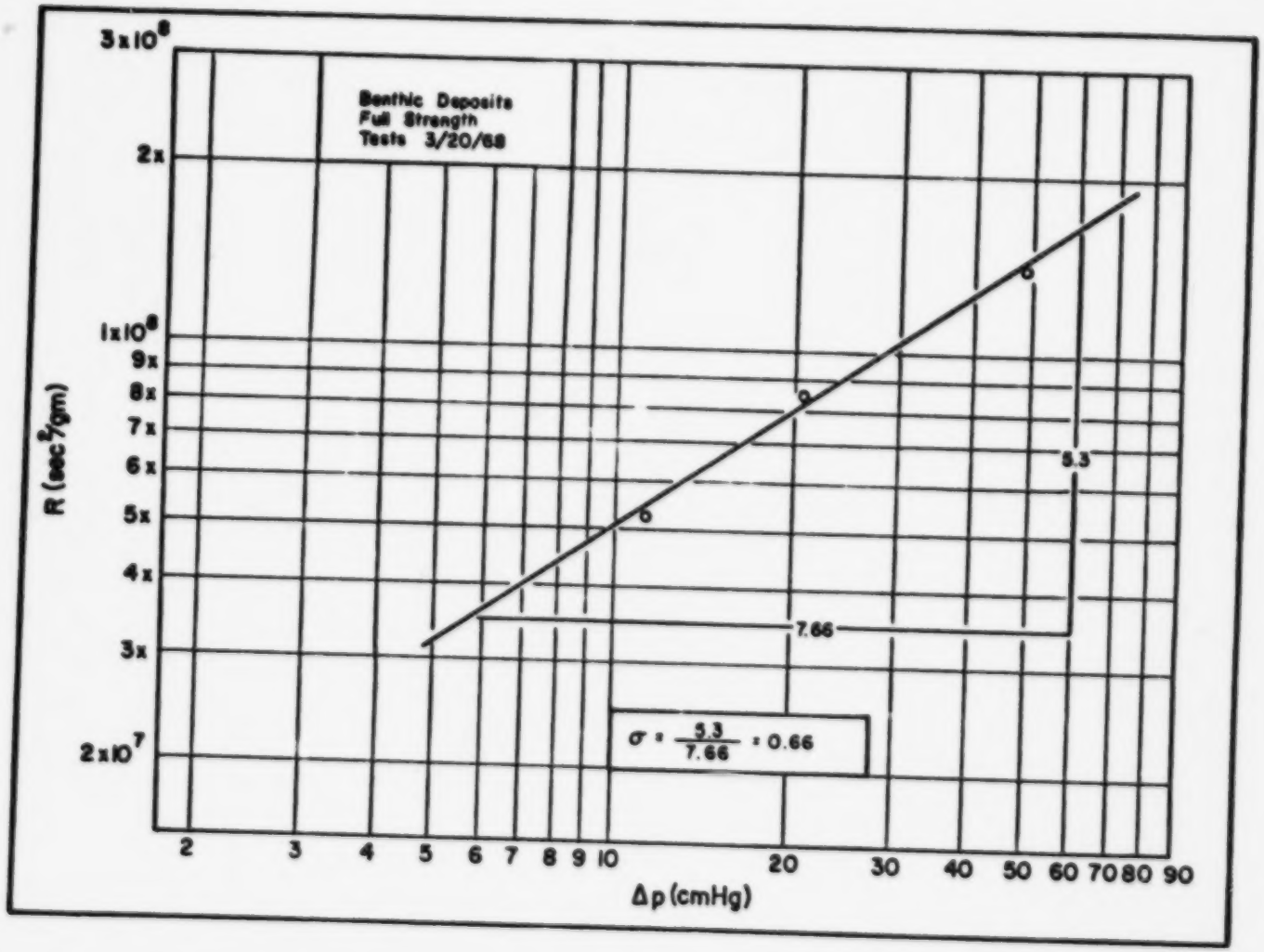

Figure 3-3: Sample plot to determine coefficient of compressibility for deposits from Campus Pond

Gravity Dewatering Apparatus

The gravity dewatering apparatus (Figure 3-4) consisted of 4 inch (10.2 cm) I. D. acrylic tubes, 36 inches (91.5 cm) in length, with flanges at both ends. Blind flanges constructed out of acrylic plate are affixed with nickel-plated nozzles. The bottom nozzle leads to the one liter graduated cylinder, by a combination of plastic and glass tubing.

The filtrate collected in the graduated (2 ml intervals) cylinder displaces air which is forced up a return tube to the top nozzle. This allows an entirely enclosed system to be maintained, preventing any measurement disturbance by evaporation. Above the bottom nozzle lies a 24 B & S gage wire mesh, which in turn supports 5 cm of coarse gravel, followed by 5 cm of A. S. T. M. standard Ottawa sand. Piezometers are attached to the column sides at appropriate intervals.

Dewatering Methodology

After the sand surfaces in the columns are carefully leveled and compacted, the sand is saturated by introduction of water from beneath. The water level is maintained at the sand surface during charging of the columns with homogenous sludge mixtures to prevent any entrapped air. It is important to note that the effective dewatering head is the distance from the sludge surface to the tip of the discharge tube located in the filtrate collection. When the piezometers are used, they are filled with water, and the rubber tubes are pinched until the columns are charged.

A pinch cock (not shown) next to the bottom nozzle is opened at $t = 0$ and volumetric readings are made at appropriate intervals. The graduated cylinders are removed, emptied, and replaced when needed, and the piezometer levels are recorded at regular intervals if used.

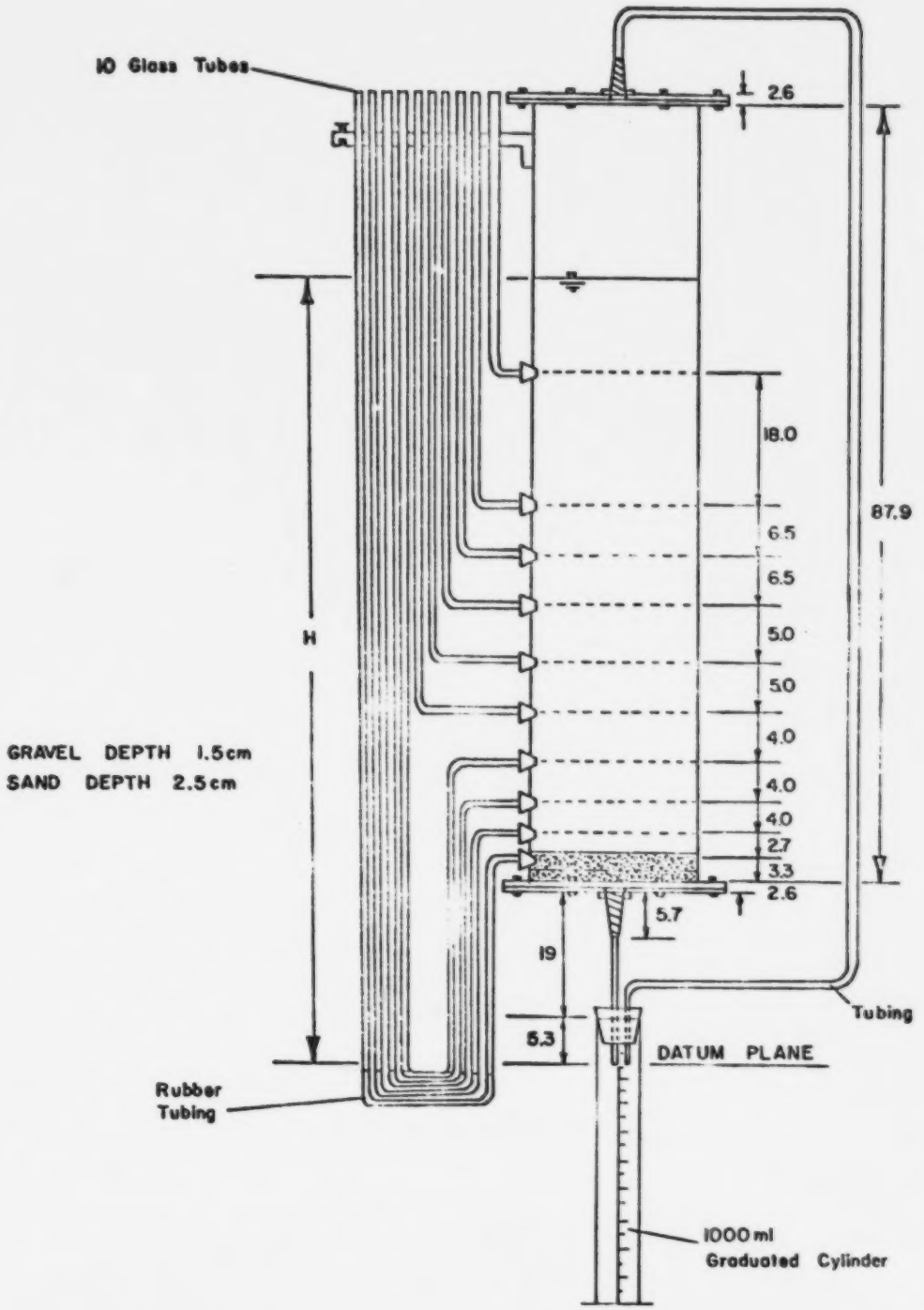

Figure 3-4: Gravity dewatering apparatus with piezometer tubes. The supporting filter media consists of 5 cm of Ottawa standard sand underlain by 5 cm of coarse gravel. Cell dimensions are in cm.

Dewatering effectively ceases upon the attainment of a pseudo-rigid plastic mass, whose surface develops menisci suspending the underlying moisture. A slow process of consolidation results which is not noteworthy for the volume of filtrate produced.

Experimental Results

Experiments conducted by the authors have shown that the headloss occuring across the sludge is predominantly in the first centimeter above the sand-sludge interface. It is clear that the sludge cake filtration theory is thus appropriate to gravity dewatering.

The volumetric readings are reduced to values of H, or total head, by subtraction and conversion of the volumes from the initial head. The values of a' and σ are determined by Buchner funnel testing. Values for ρ, g and μ are, of course, known. Hence, the only factor remaining to be determined in Equation 2-19 is the value of c, and this can be calculated from Equation 3-3a.

Sample results for a wastewater sludge using Equation 2-19 is shown in Figure 3-5. The initial depth of sludge of 24 cm (note: not the head H) at a wet-basis water content of 96.3% was reduced to a depth of 15 cm.

Three identical samples dewatered almost identically, although one in the latter stages demonstrated a sharp deviation due to a phenomena of solids break-through into the sands. This was most likely due to an oversight in the filling of the sand on top of the gravel, leaving too shallow a layer of sand to support the sludge and the viscous shear of the filtrate.

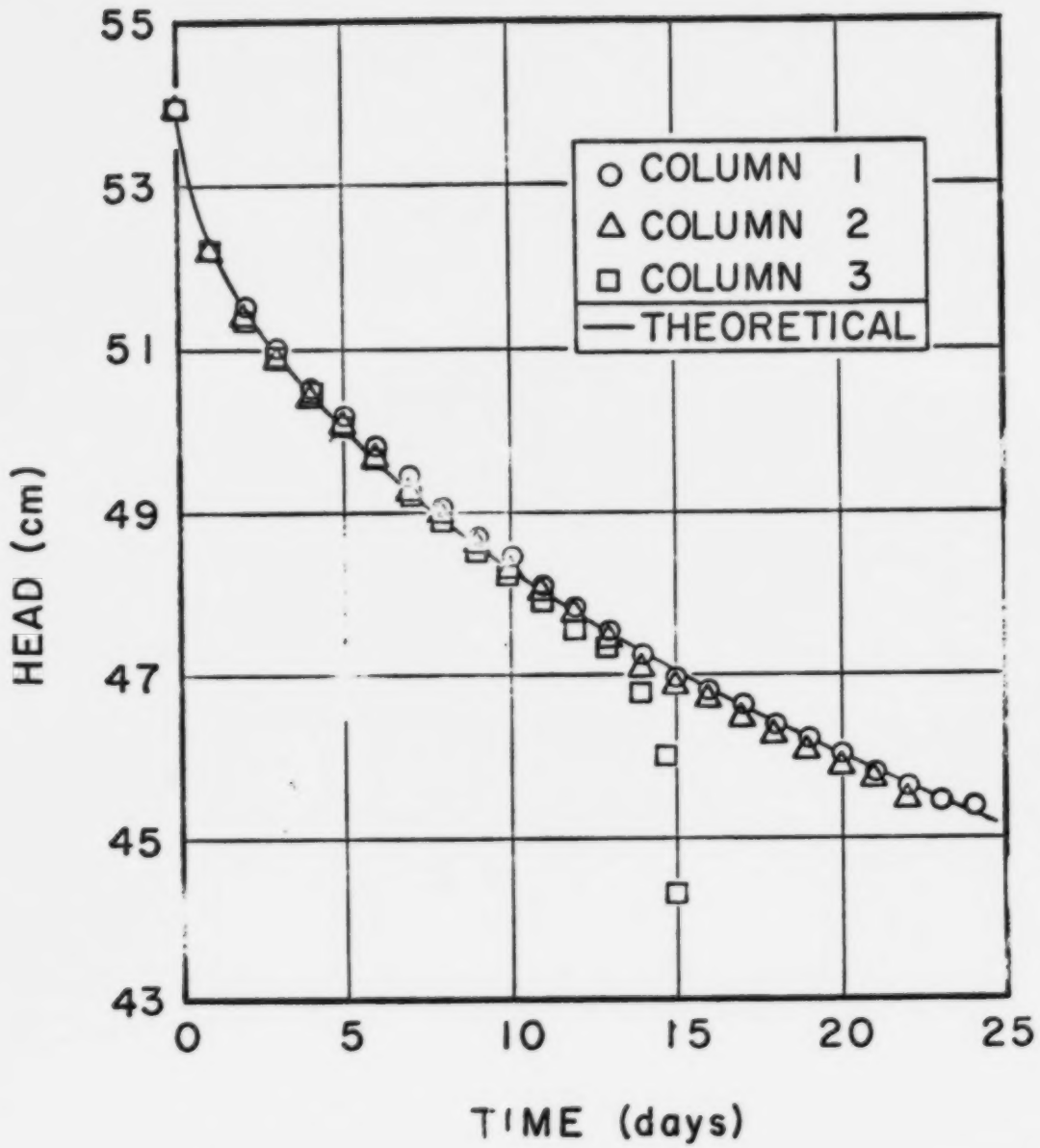

Figure 3-5: Drainage of a wastewater sludge as a function of time. The theoretical curve refers to Equation 2-19.

IV. RESULTS AND CONCLUSIONS

Reversal of the aging process of a eutrophic lake, reservoir or estuary will require removal of the benthic deposits. These deposits serve as a reservoir for the nutrients nitrogen and phosphorous which must be present in solution to promote algal blooms and subsequent eutrophication. Land disposal of the dredged solids will be difficult unless volume and weight reduction can be achieved through a dewatering step, for it is highly likely that as much as 90-95% of the weight of dredged material will be water.

Dewatering on soils such as sand is hampered by the near absence of design data with which to design such a facility. A mathematical model applicable to such design has been developed and verified with laboratory column tests. It is dependent upon three parameters; solids content, specific resistance and coefficient of compressibility; all of which are readily measured through laboratory tests.

The parameters specific resistance and coefficient of compressibility were measured in the laboratory for three benthic deposits for the first time for three Massachusetts lakes. Additional data is needed for a wide spectrum of benthic deposits in order to provide design data for field evaluation of this dewatering process.

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